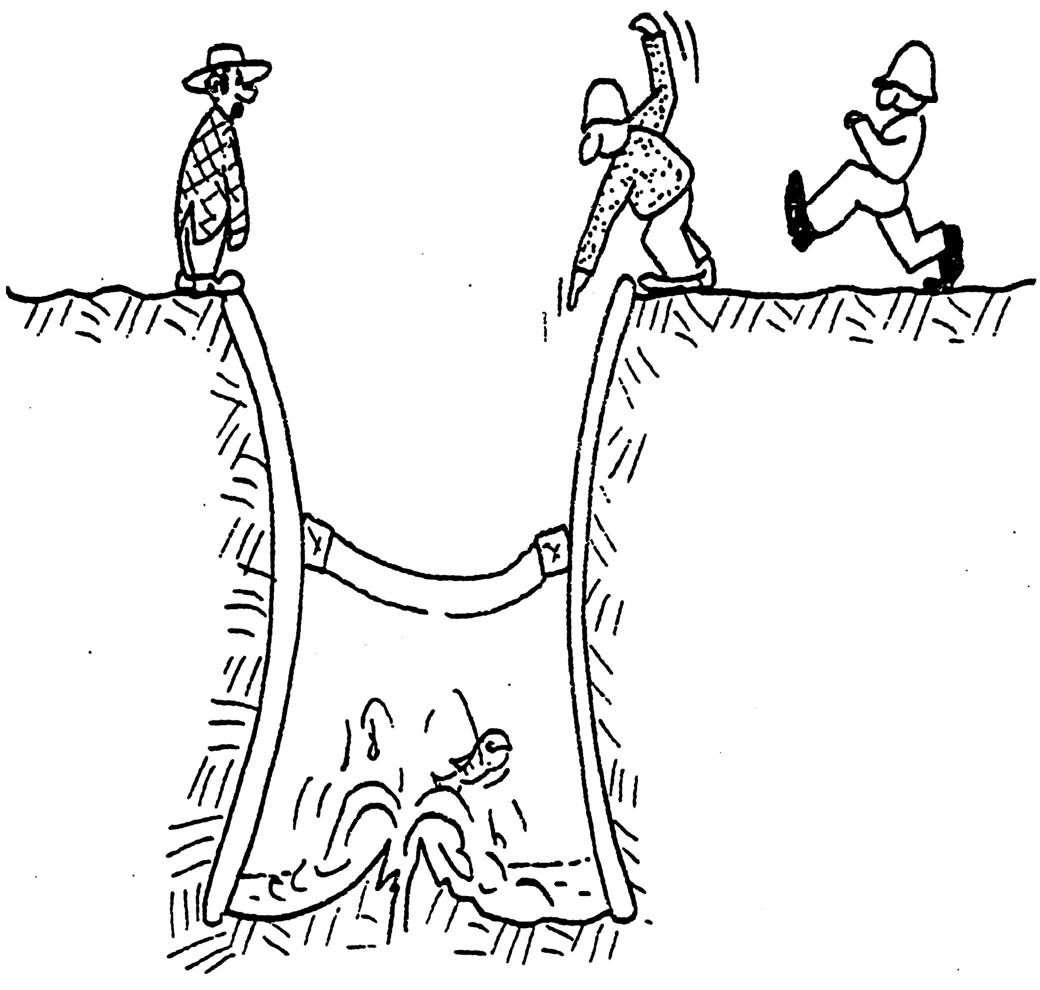
SPECIAL CONDITIONS



SPECIAL CONDITIONS

SPECIAL CONDITIONS

The best shoring system in the world would be of little value if the soil being supported does not act as contemplated by the designer. Adverse soil properties and changing conditions need to be considered.

Anchors placed within a soil failure wedge will exhibit little holding value when soil movement in the active zone occurs. The same reasoning holds for the anchors or piles in soils which decrease bonding or shear resistance due to changes in plasticity or cohesion. Additional information regarding anchors may be found in the USS Steel Sheet Piling Design Manual.

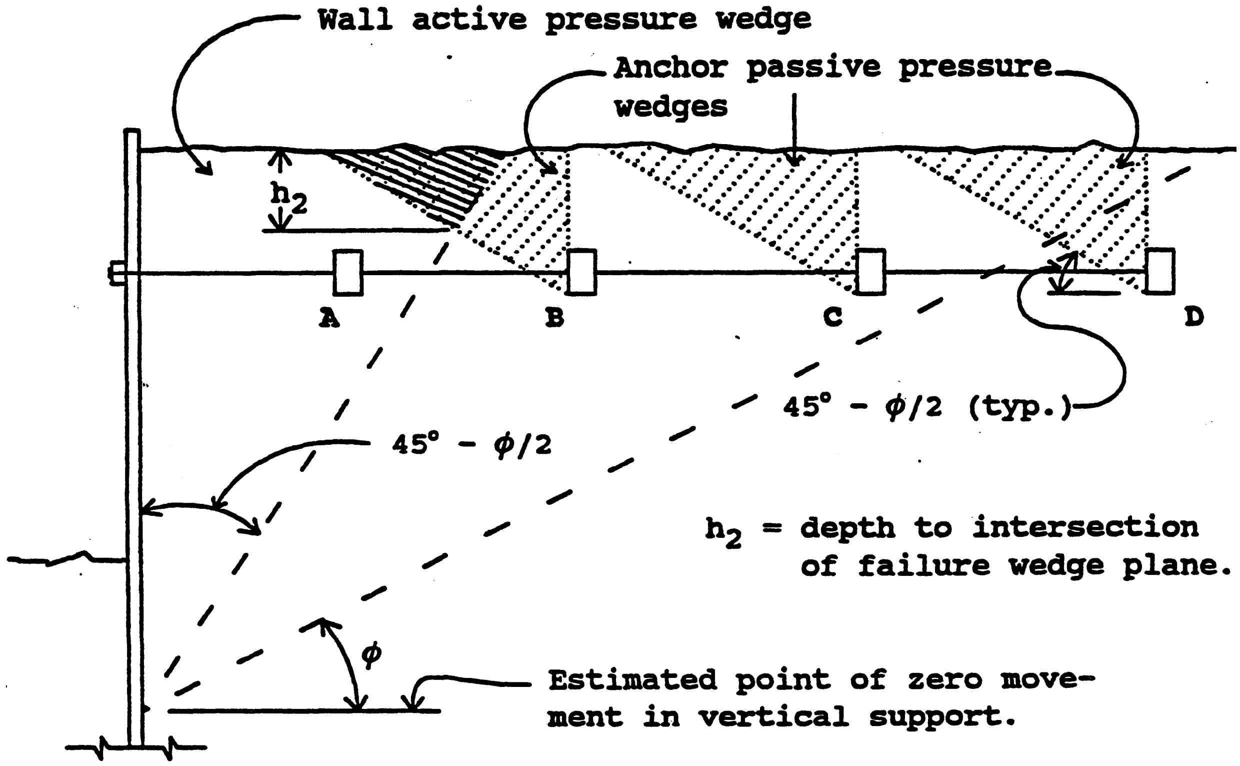
When cohesive soils tend to expand or are pushed upward in an excavation, the shoring wants to move laterally. Soil rising in an excavation indicates that somewhere else soil is settling. Water rising in an excavation can lead to quick conditions, while water moving horizontally can transport soil particles leaving unwanted voids at possibly critical locations.

Avery important consideration always present in all but a few types of shoring systems is the potential for a sudden failure due to slippage of the soil around the shoring system along a surface offering the least amount of resistance.

Sample situations of the above are included on the following pages.

DEADMEN

The size, shape, depth and location of an anchor block affects the resistance capacity developed by that anchor. The following diagram explains how the distance from the wall affects capacity.



Deadman A located inside active wedge and offers no resistance.
Deadman B resistance is reduced due to overlap of the active wedge (wall) and the passive wedge (anchor).

Anchor reduction: (Granular soils)

$$\Delta P_p = (1/2)(K_p - K_a)\gamma h_2^2$$

ΔP_p is transferred to the wall.

Deadman C develops full capacity but increases pressure on wall
Deadman D develops full capacity and has no effect on bulkhead.

Deadmen should be placed against firm natural soil and should not be allowed to settle.

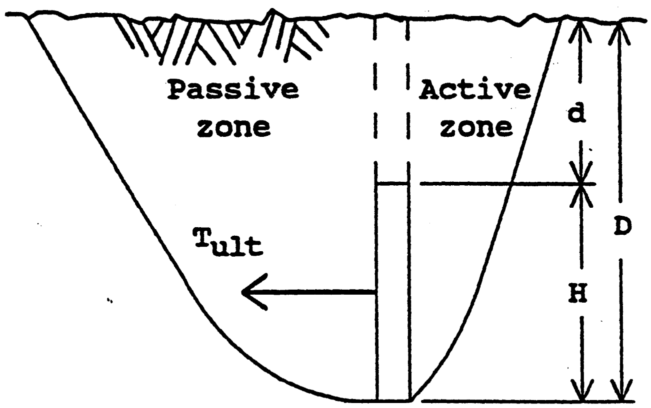
A safety factor of 2 is recommended for all anchors and deadmen.

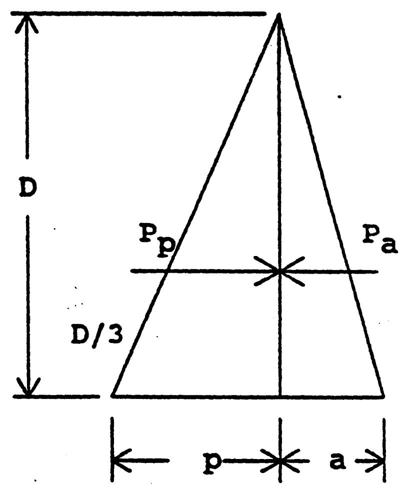
The following criteria is for anchors or deadman located entirely in the passive zone as indicated by Anchor D.

SPECIAL CONDITIONS

DEADMEN IN COHESIONLESS SOIL NEAR GROUND SURFACE :$d \le H/2$.

The forces acting on an anchor are shown in the following diagrams. For this case, $d \le H/2$, it is assumed that the anchor extends to the ground surface.





GRANULAR SOILS

where $p = \gamma D K_p$; $a = \gamma D K_a$

The capacity of a deadman also depends on whether it is continuous (long) or short. A deadman is considered continuous when its length greatly exceeds it height.

The basic equation is:

$T_{ult} = L(P_p - P_a)$ where L = Length of Deadman.

For continuous Deadmen

$P_a = K_a \gamma D^2/2$ $\qquad P_p = K_p \gamma D^2/2$

$T_{ult} = \gamma D^2 (K_p - K_a) L\}/2$

For Short Deadmen ($L \le 3H$)

$T_{ult} = L(P_p - P_a) + (\gamma K_o D^3 \tan\phi [K_p + K_a]^{1/2})/3$

$K_o = 0.4$ is recommended.

DEADMEN IN COHESIONLESS SOIL where $1.5 \leq D/H \leq 5.5$

This chart is based on sand of medium density, (ϕ= 32.5). For other values of ϕ a linear correlation may be made from (ϕ /32.5). The chart is valid for ratios of depth to height of anchor (D/H) between 1.5. and 5.5.

For square deadman the value from the chart (K_p') is larger than the value for continuous deadman (KP). This is because the failure surface is larger than the actual dimensions of the deadman. In testing it is determined to be approximately twice the width.

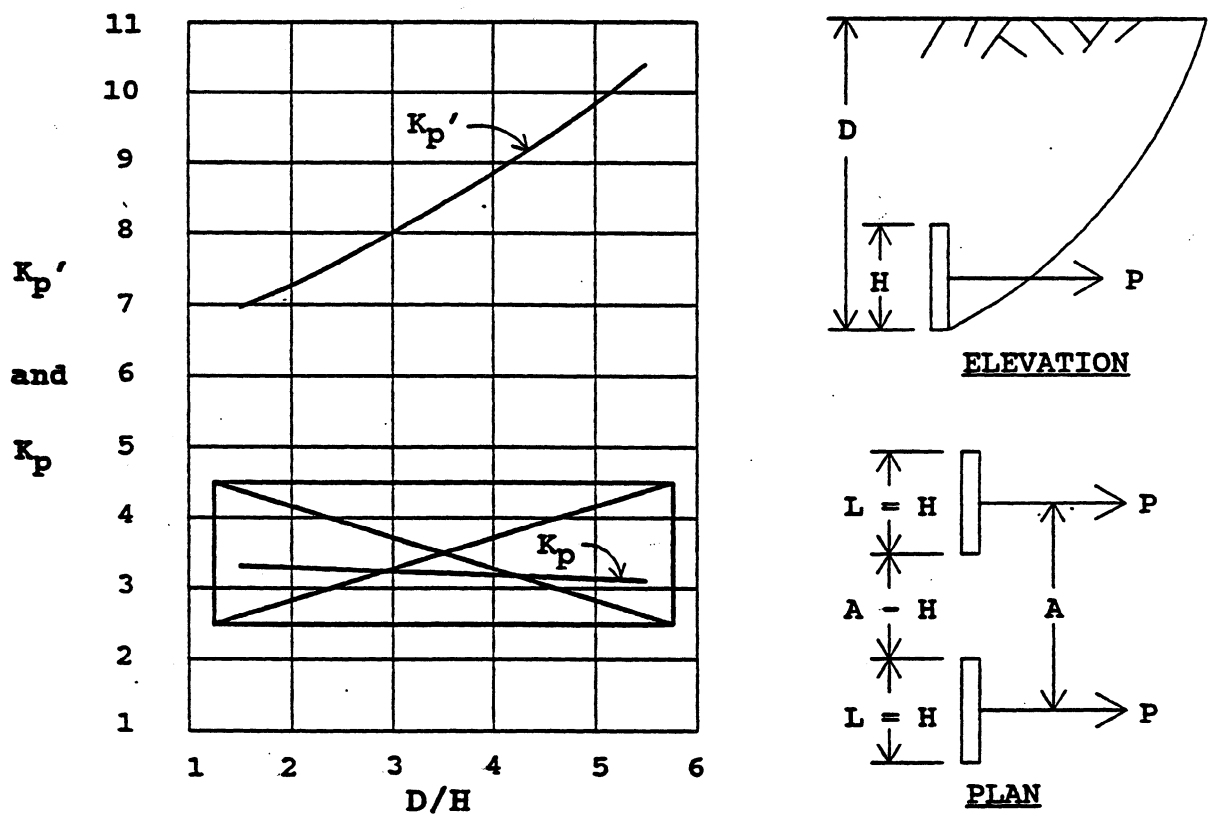


FIGURE 23

For Continuous Deadman

Use Ovesen's method as described in the USS Steel Sheet Piling Design Manual.

For Square (or Short) Deadmen, L = H

$$P_{ult} = (\gamma H^2 K_p' L)/2$$

It is recommended that a factor of safety of 2 be used.

DEADMEN IN COHESIONLESS SOIL where $D/H \geq 5.5$

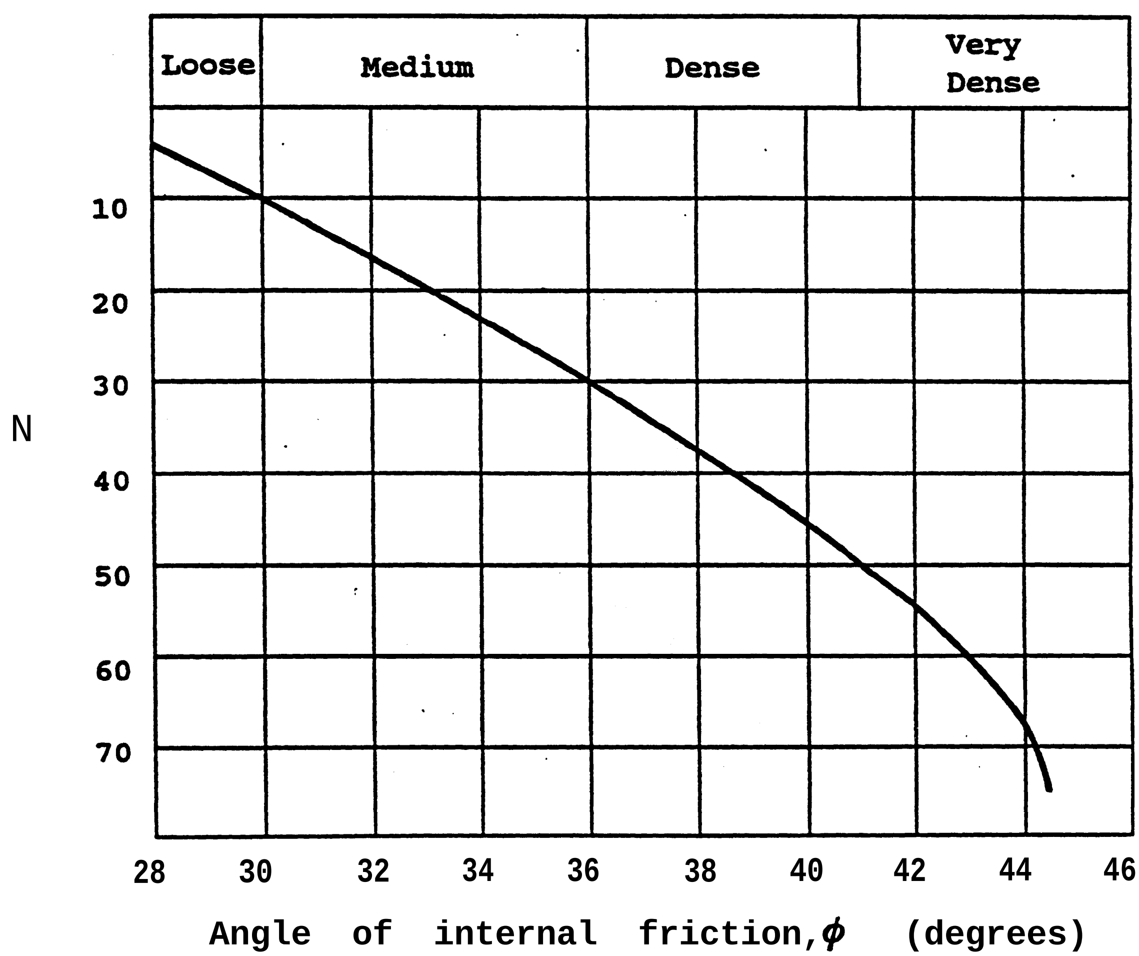
When deadmen are placed at great depth the resistance can be approximately calculated as the capacity of a footing at a depth equal to the center of the deadman. Resistance can also be estimated from the following equations and chart.

T_{ult} = Bearing capacity of a footing at a depth equal to D + H/2 (see page 11-3).

Where water is not a factor:

$$T_{ult} \text{ (Square Block)} = 2LN^2 + 6H(100 + N^2)(LH)$$

$$T_{ult} \text{ (Long Block)} = 3LN^2 + 5H(100 + N^2)(LH)$$

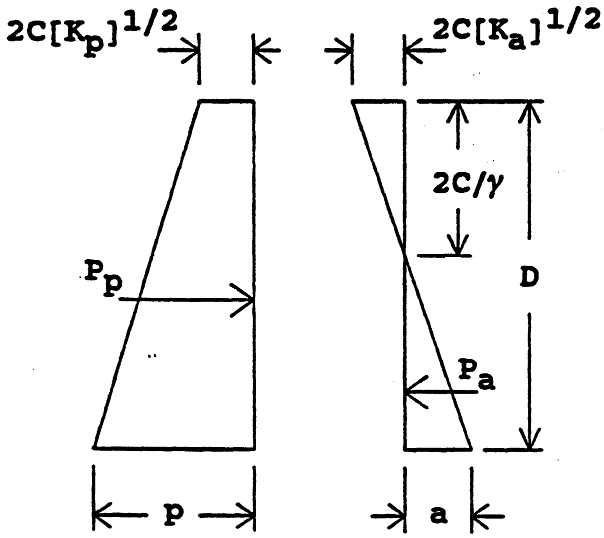


N = Standard Penetration Resistance (Number of blows per foot)

FIGURE 24

DEADMEN IN COHESIVE SOIL NEAR THE GROUND SURFACE $d \le H/2$

The forces acting on an anchor are shown in the following diagrams. For this case, $d \le H/2$ (see page 11-3), it is assumed that the anchor extends to the ground surface, Capacity of the anchor depends upon whether it is considered continuous or short.



COHESIVE SOIL

where $p = \gamma D K_p + 2C[K_p]^{1/2}$
$a = \gamma D K_a - 2C[K_a]^{1/2}$

The pressure diagram for cohesive soils assumes a short load duration. For a duration of a period of years it is likely that creep will change the pressure diagram. Therefore conservative assumptions should be used in the analysis, such as:

$C = 0$ and $\phi = 27°$

The basic equation is:

$T_{ult} = L(P_p - P_a)$ where L = Length of Deadman.

For Continuous Deadmen:

$$P_p = \gamma D^2 K_p/2 + 2CD[K_p]^{1/2}$$

$$P_a = (\gamma D K_a - 2C[K_a]^{1/2})(D - 2C/\gamma)/2$$

It is recommended that the tension zone be neglected.

$$T_{ult} = L(P_p - P_a$$

For Short Deadmen:

$$T_{ult} = L(P_p - P_a) + 2CD^2$$

DEADMEN IN COHESIVE SOIL where d $\geq$ H/2

A chart has been developed through testing for deadmen other than near the surface. This chart relates a dimensionless coefficient (R) to the ratios of depth to height of an anchor (D/H) to determine the capacity of the deadman.

This chart applies to continuous anchors only.

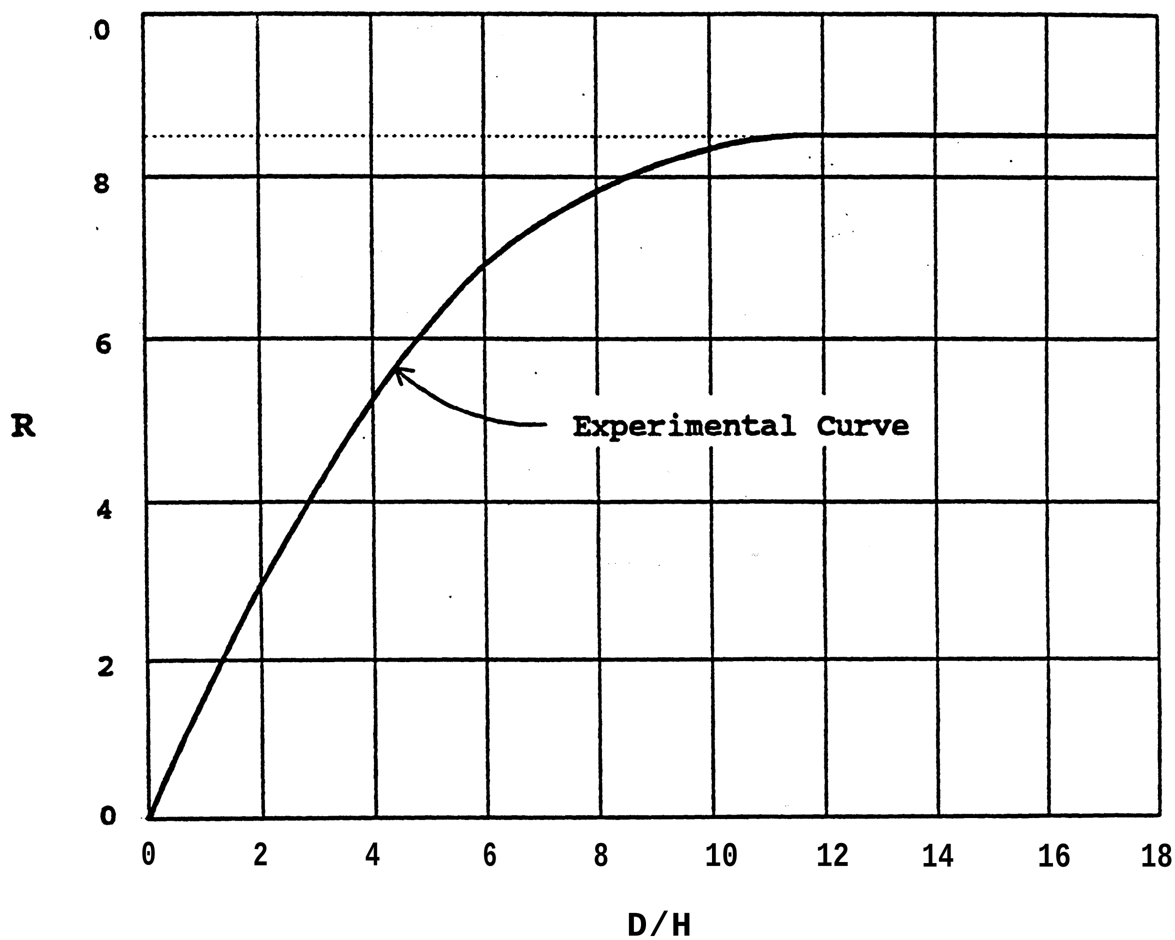


FIGURE 25

The above graph is from Strength of Deadmen Anchors in Clay, Thomas R. Mackenzie, Master's Thesis Princeton University, Princeton, New Jersey, 1955.

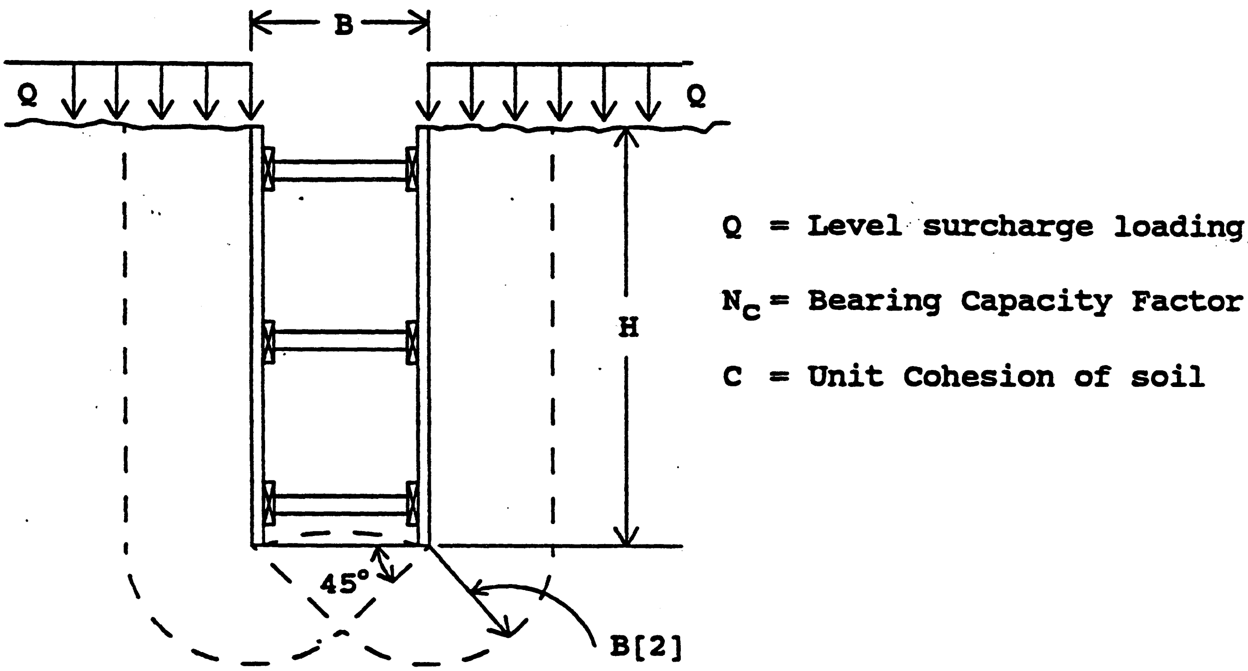
P_{ult} = RCHL with a maximum value of R = 8.5.

It is recommended that a factor of safety of 2 be used.

HEAVE

The condition of heave can occur in soft plastic clays when the depth of the excavation is sufficient to cause the surrounding clay soil to displace vertically with a corresponding upward movement of the material in the bottom of the excavation.

The possibility of heave and slip circle failure in soft clays, and in the underlying clay layers, should be checked when the Stability Number (N_o) exceeds 6 (Stability Number, $N_o = \gamma H/C$).



The relative layout of the excavation influences how heave may be checked. The two conditions depend on whether the sides of the excavation are in close proximity of each other as comparedto the depth.

For the condition of H < B (wide, shallow excavations)

Critical Height $H_C = (5.7C - Q)/\{\gamma - (C/B)[2]^{1/2}\}$ (Terzaghi)

For the condition of H > B (trench type excavations)

critical Height $H_C = (CN_c - Q)/\gamma$ (Skempton)

The Bearing capacity factor, N_C, is determined from FIGURE 26 on the following page.

It is recommended that a minimum safety factor of 1.5 be applied to we Unit Cohesion of soil (use C/1.5).

If heave is probable while using the minimum safety factor it could be prevented by extending the shoring system (sheeting) below the bottom of the excavation into a more stable layer, or for a distance of one-half the width of the excavation (typically valid for only excavations where H>B). Another solution would be overexcavating and constructing a counterweight or tremie seal.

NOTE- Strutting a wall near its bottom will not prevent heave but such strutting may prevent the wall from rotating into the excavation.

A procedure for calculating the critical height H_c, at which heaving could occur is outlined on the previous page.

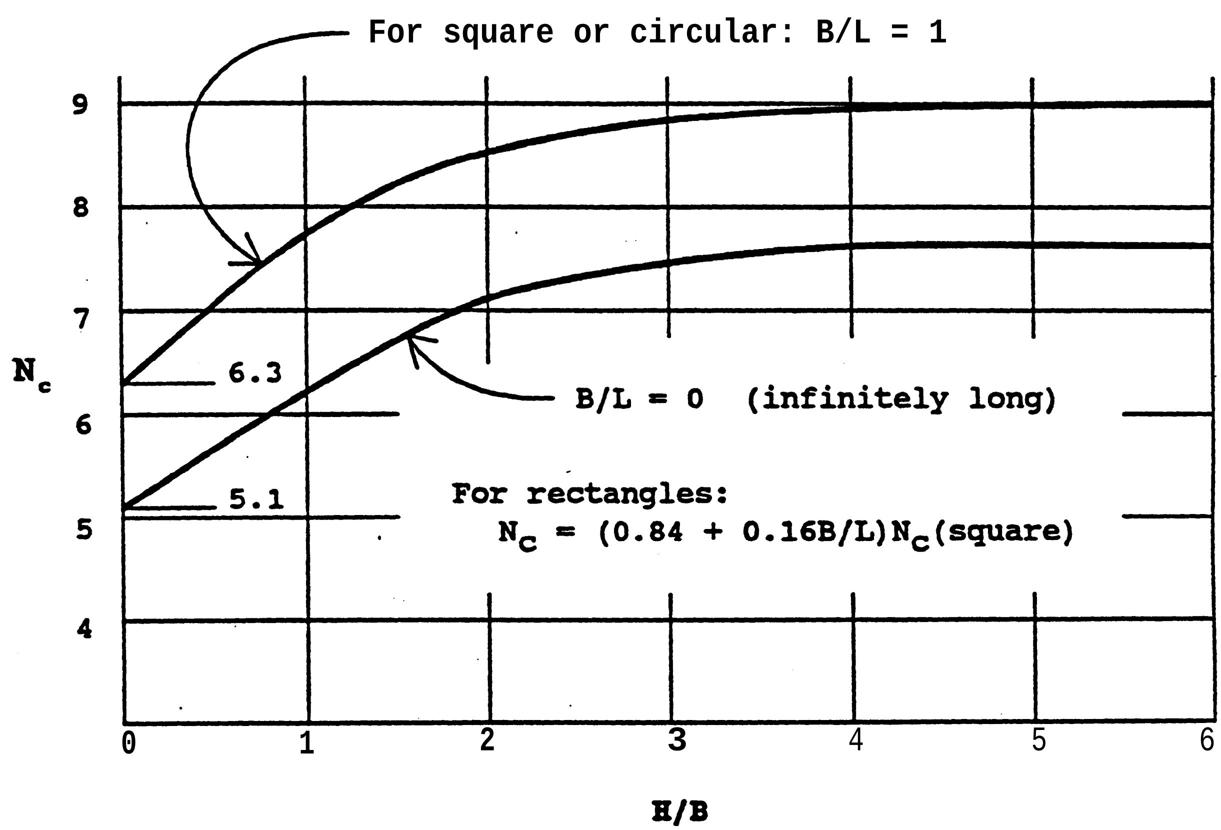


Diagram for determining the Bearing Capacity Factor, N_C

FIGURE 26

EXAMPLE PROBLEM for $H < B$

Given: $H = 12'$, $B = 20'$, $L = 40'$
$Q = 300$ psf, $C = 400$ psf, $\gamma = 115$ pcf

Solution:
$$H_c = (5.7C - Q)/\{\gamma - (C/B)[2]^{1/2}\}$$
$$= \{5.7(400) - 300\}/\{115 - (400/20)(1.414)\}$$
$$= 22.8' > 12'$$

If we were to apply the minimum recommended Safety Factor of 1.5, the value of C to use would be:

$C/1.5 = 267$ psf.

$$H_c = \{5.7(267) - 300\}/\{115 - (267/20)(1.414)\}$$
$$= 12.7' > 12'$$

Since H is less than H_c with a Safety Factor- considered, heave is not expected to occur.

EXAMPLE PROBLEM for $H > B$

Given: $H = 25'$, $B = 10'$, $L = 40'$
$Q = 300$ psf, $C = 400$ psf, $\gamma = 115$ pcf

Solution: $H/B = 25/10 = 2.5$, $B/L = 10/40 = 0.25$

From FIGURE 26 on the previous page,

N_c square = 8.8

For rectangle:

$$N_c = \{0.84 + 0.16(0.25)\}8.8 = 7.74$$

Now applying the Safety Factor of 1.5 to the Cohesive value; $C = 400/1.5 = 267$ psf.

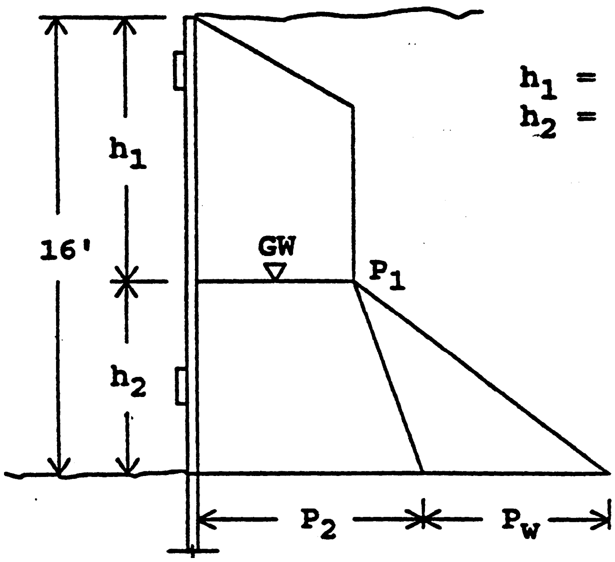
$$H_c = (CN_c - Q)/\gamma$$
$$(267(7.74) - 300)/115$$
$$15.4' < 25'$$

Since $H_c < H$, heave is likely to occur. For this case extending the shoring +5' deeper should be considered.

SPECIAL CONDITIONS

GROUND WATER

EXAMPLE: Soldier Piles w/Struts (restrained system)



$h_1 = 10'$
$h_2 = 6'$

From Soils Report

$\gamma = 110$ pcf
$\gamma_b = 0.6\gamma$
$= 66$ pcf
$Kw = 38$ pcf

$Kw = K_a\gamma$
$K_a = Kw/\gamma = 38/110$
$= 0.345$

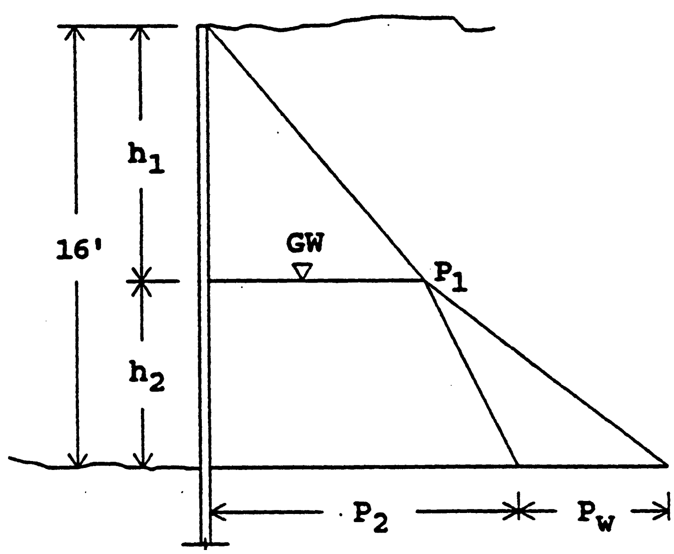
$P_1 = 0.71K_a\gamma h_1 = 0.71Kwh_1$

$P_1 = (0.71)(38)(10)$
$= 270$ psf

$P_2 = P_1 + 0.71K_a\gamma_b h_2 = 270 + (0.71)(0.345)(66)(6) = 367$ psf

$P_W = (62.4)(6) = 374$ psf

EXAMPLE: Sheet Piling (cantilevered system)



Same conditions as above

$Kw = K_a\gamma$
$K_a = Kw/\gamma = 38/110$
$= 0.345$

$P_1 = K_a\gamma h_1 = Kwh_1$

$P_1 = (38)(10)$
$= 380$ psf

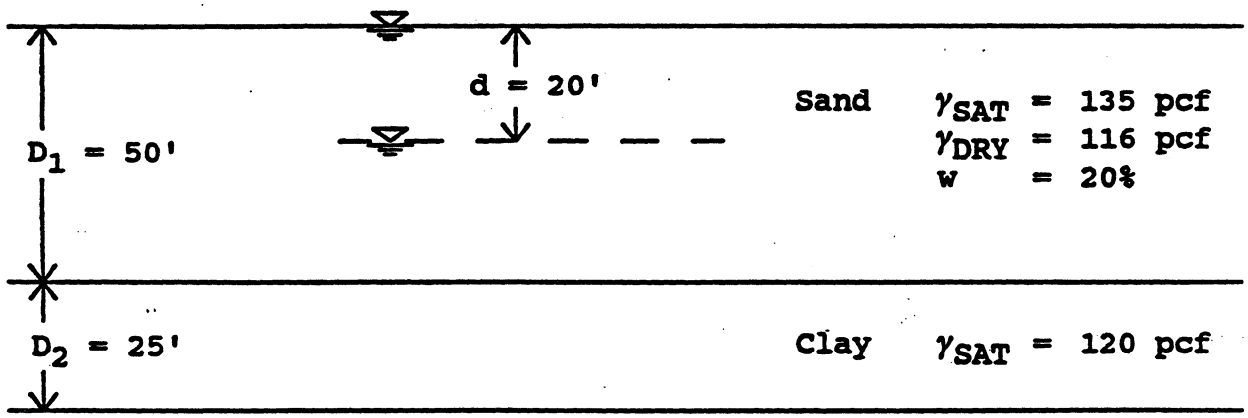
$P_2 = P_1 + K_a\gamma_b h_2 = 380 + (0.345)(66)(6) = 517$ psf

$P_w = (62.4)(6) = 374$ psf

LOWERED WATER TABLE

When the water table is lowered more load is transmitted to the underlying soil. This is due to the fact that the water's bouyant force on the soil particles is removed when the water table is lowered.

EXAMPLE PROBLEM



Given: Water surface originally at ground surface.
Water surface is lowered 20 Ft.

Find: Pressure at the center of the clay layer.

Solution:

Initial

$$P = D_1(\gamma_{SAT} - \gamma_{WATER}) + (D_2/2)(\gamma_{SAT} - \gamma_{WATER})$$

$$= 50(135 - 62.5) + (25/2)(120 - 62.5) = \quad 4344 \text{ psf}$$

Lowered

$$P = d\{(\gamma_{DRY} + 0.2(\gamma_{SAT} - \gamma_{DRY})\} + (D_1 - d)(\gamma_{SAT} - \gamma_{WATER}) + (D_2/2)(\gamma_{SAT} - \gamma_{WATER})$$

$$= 20\{116 + 0.2(135 - 116)\} + 30(135 - 62.5) + 12.5(120 - 62.5)$$
$$= 5290 \text{ psf}$$

$$\Delta P = 5290 - 4344 = 946 \text{ psf increase}$$

Check: $\Delta P = 20\{62.5 - 0.8(135 - 116)\} = 946$ psf

SPECIAL CONDITIONS

PIPING

For excavation in pervious materials (sands), the condition of piping can occur when an unbalanced hydrostatic head exists. This causes large upward flows of water through the soil and into the bottom of the excavation. Material will be transported, which, if allowed to continue, will cause settlement of the soil adjacent to the excavation. This is also known as a sand boil or a quick condition. The passive resistance of embedded members will be reduced.

To correct this problem, either equalize the unbalanced hydraulic head by allowing the excavation to fill with water or lower the water table outside the excavation by dewatering.

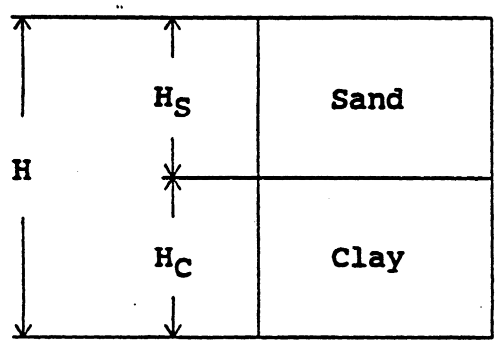
If the embedded length of the shoring system member is long enough, the condition of piping should not develop. Charts giving lengths of sheet pile embedment which will result in an adequate factor of safety against piping shown on page 65 of the USS Steel Sheet Piling Design Manual. These charts are of particular interest for cofferdams constructed of sheet piling.

STRATIFIED LAYER CONVERSION

Mixed layers of soil can be converted to an approximate: equivalent: clay type soil by use of the equations given below. These equations merely convert the soil to an equivalent clay based on the weighted averages of the individual layers. Note that this approximationmay result in a total horizontal pressure which is less than that calculated by the trial wedge method.

Do not use this method when there is a clay layer on top. In this situationmake a separate calculation for the top layer, then use an equivalent soil for the remainder of the depth. Another acceptable method which will take care of any situation is the semi-graphical trial wedge.

Case I (sand over clay)



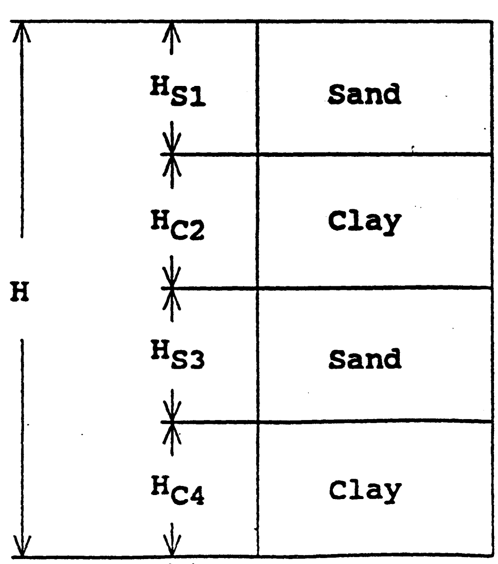
Equivalent Clay

$$q = (K_S \gamma_S H_S^2 \tan \phi + H_C n q_u)/H$$

$$\gamma = (\gamma_S H_S + H_C \gamma_C)/H$$

(See Note Below)

Case II (multiple layers, sand on top)



Use $H' = K_S H_S^2 \tan \phi$

$$q = [\gamma_1 H'_1 + \gamma_2 H_{C2} + \gamma_3 H'_3 + \gamma_4 H_{C4}]/H$$

$$\gamma = [\gamma_1 H_{S1} + \gamma_2 H_{C2} + \gamma_3 H_{S3} + \gamma_4 H_{C4}]/H$$

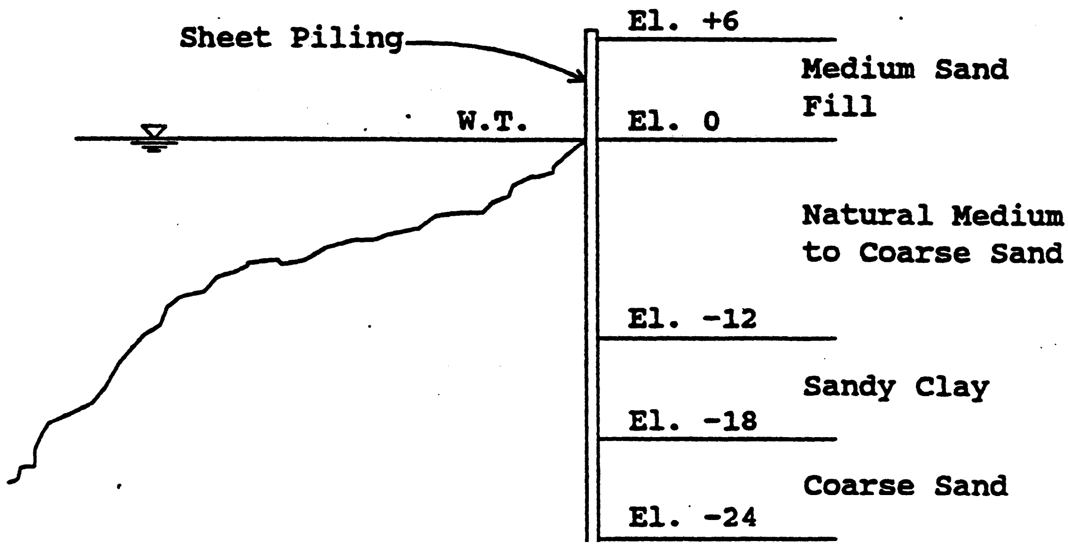
Note: For practical purposes K_S and n can be assumed to be equal to 1.0.

Subscripts S and C refer to sand and clay.

For sand $q_u = 0$

EXAMPLE PROBLEM - SHEET PILING USING STRATIFIED SOIL.

This sheet piling problem shows the effect on horizontal pressure after the sandy clay layer is drained; see inital and final pressure diagrams. Two pressures exist at each soil boundary elevationbecause a different K value is used for each soil type.



Assume

G = 2.65 for all soils.

P_V = Vertical Pressure

P_A = Active Horizontal Pressure

$P_A = P_V \tan^2(45° - \phi/2) - 2C\tan(45° - \phi/2)$

$\gamma_{eff} = (1 + w)\gamma dry$

Void Ratio = $e = (G\gamma_w/\gamma_{dry}) - 1$, $\gamma_w = 62.4$

W = water content

Layer	γ_{dry}	w	γ_{eff}	$\phi°$	e	C
Sand Fill:	100	10.0%	110	28		
Natural Sand:	102	24.5%	127	28	0.62	
Sandy Clay:	83	37.0%	114	16	0.99	275
Coarse Sand:	105	21.6%	128	36	0.575	

Submerged Weight = $\gamma_{SUB} = (G - 1)\gamma_w/(1 + e)$

Natural Sand: $\gamma_{SUB} = (1.65)(62.4)/(1 + 0.62) = 63.6$ pcf

Sandy Clay: $\gamma_{SUB} = (1.65)(62.4)/(1 + 0.99) = 51.7$ pcf

Coarse Sand: $\gamma_{SUB} = (1.65)(62.4)/(1 + 0.575) = 65.4$ pcf

CALCULATION OF PRESSURES

No Fill Condition (See diagram A on next sheet)

El. 0 $P_A = 0$ psf
El. -12 $P_A = 12(63.6)\tan^2(45^\circ - 28^\circ/2) = 276$ psf

El. -12 $P_A = 12(63.6)\tan^2(45^\circ - 16^\circ/2) - 2(275)\tan(45^\circ - 16^\circ/2)$
$= 19$ psf
El. -18 $P_A = \{12(63.6) + 6(51.7)\}\tan^2(45^\circ - 16^\circ/2)$
$- 2(275)\tan(45^\circ - 16^\circ/2)$
$= 195$ psf

El. -18 $P_A = \{12(63.6) + 6(51.7)\}\tan^2(45^\circ - 36^\circ/2) = 279$ psf
El. -24 $P_A = \{12(63.6) + 6(51.7) + 6(65.4)\}\tan^2(45^\circ - 36^\circ/2)$
$= 381$ psf

Drained Condition (See diagram B on next sheet)

El. +6 $P_A = 0$ psf
El. 0 $P_A = 6(110)\tan^2(45^\circ - 28^\circ/2) = 238$ psf

El. 0 $P_A = 6(110)\tan^2(45^\circ - 28^\circ/2) = 238$ psf
El. -12 $P_A = \{6(110) + 12(63.6)\}\tan^2(45^\circ - 28^\circ/2) = 514$ psf

El. -12 $P_A = \{6(110) + 12(63.6)\}\tan^2(45^\circ - 16^\circ/2) -$
$2(275)\tan(45^\circ - 16^\circ/2)$
$= 394$ psf
El. -18 $P_A = \{6(110) + 12(63.6) + 6(51.7)\}\tan^2(45^\circ - 16^\circ/2)$
$- 2(275)\tan(45^\circ - 16^\circ/2)$
$= 570$ psf

El. -18 $P_A = \{6(110) + 12(63.6) + 6(51.7)\}\tan^2(45^\circ - 36^\circ/2)$
$= 450$ psf
El. -24 $P_A = \{6(110) + 12(63.6) + 6(51.7) + 6(65.4)\}$
$\tan^2(45^\circ - 36^\circ/2)$
$= 552$ psf

Immediately after the sand fill is placed an increase of 660 PSF overburden pressure (vertical) is applied to the soil and the water contained therein. For granular soils the increased water pressure quickly dissipates. Cohesive soils do not allow free flow of water, therefore for some period of time the pressure acting horizontally is equal to the vertical pressure, See diagram c on the next page.

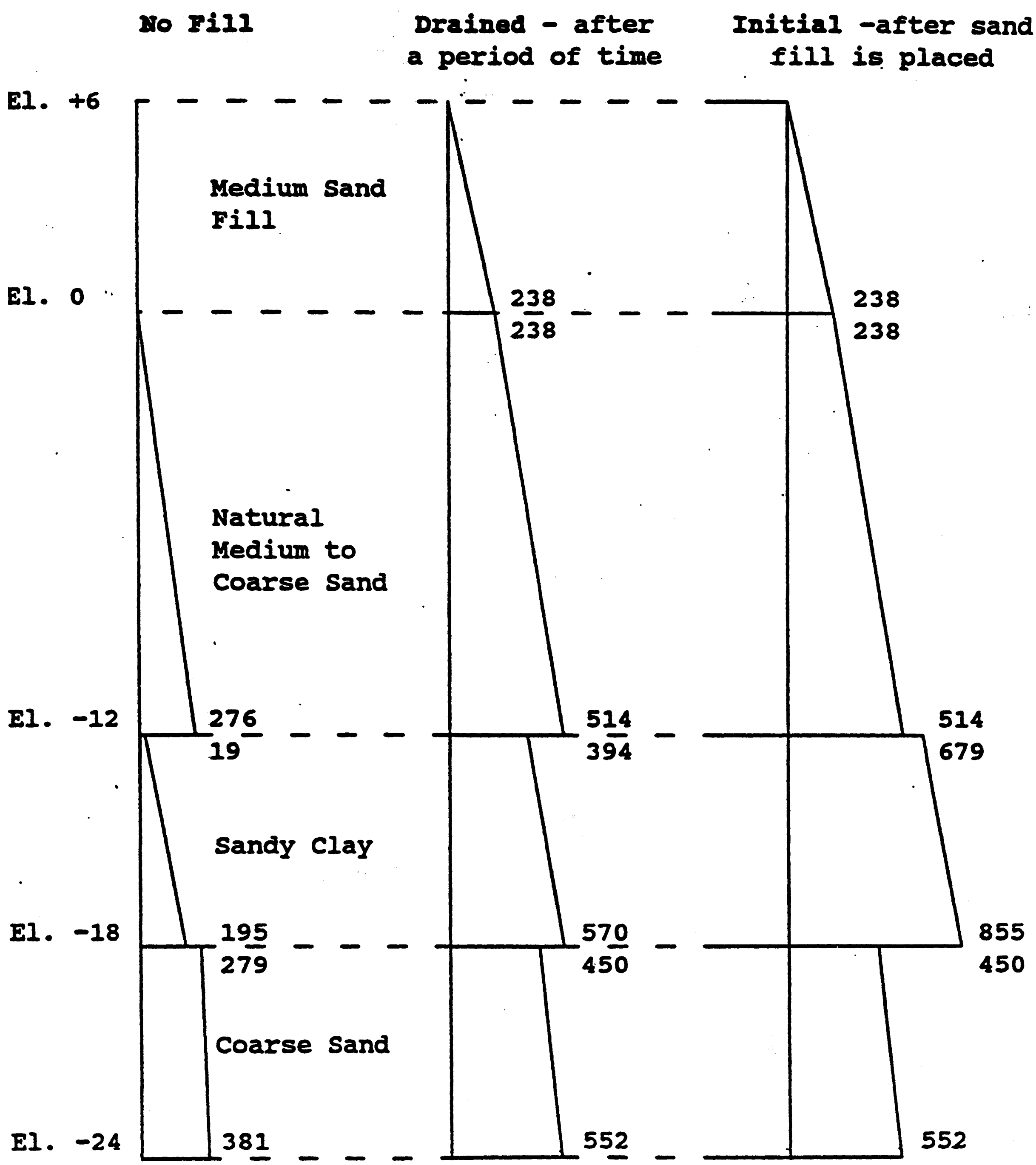
SPECIAL CONDITIONS

Initial Fill Condition (diagram, C)

Pressure in Clay Layer. Immediatly After Sand Fill is Placed.

El. −12 P_A = 6(110) + 19 = 679 psf

El. −18 P_A = 6(110) + 195 = 855 psf



Diagram

SLOPE STABILITY

The most critical failure surface will be dependent on site geology and is not necessarily circular. Non-circular failure surfaces can be caused by adversely dipping bedding planes, zones of weak soil or unfavorable ground water conditions. Circular solutions to slope stability have been developed primarily because of the ease this geometry lends to the computational procedure.

Two approximate methods used for investigating the factor of safety for potential stability failure are:

'Fellenius Method of Slices'
'Simplified Bishop Method of Slices'

The basic equation for each of these methods is:

$$F = \{\bar{C}\,L + \tan\bar{\phi}\sum_{i=1}^{i=n}\bar{N}_i\}/\{\sum_{i=1}^{i=n}W_i\sin\theta_i\}$$

Nomehclature

F = Factor of safety

F_a = Assumed factor of safety

i = Represents the current slice

$\bar{\phi}$ = Friction angle based on effective stresses

C = Cohesion intercept based on effective stresses

W_i = Weight of the slice

Ni = Effective normal force

θ_i = Angle from the horizontal of a tangent at the center of the slice along the slip surface

T_i = Tensile force

u_i = Pore-water pressure force on a slice

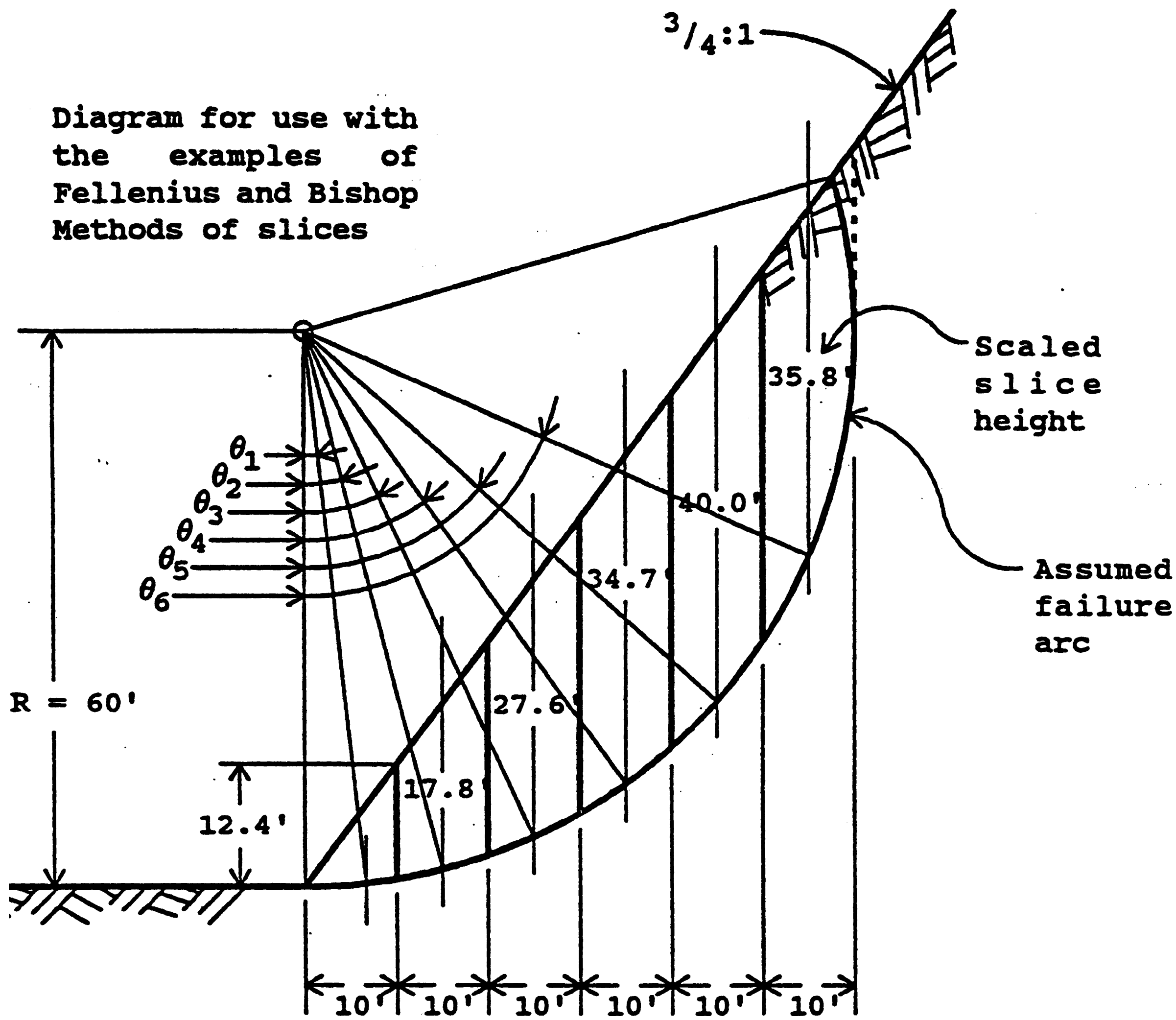
u_i = Resultant neutral (pore-water pressure) force

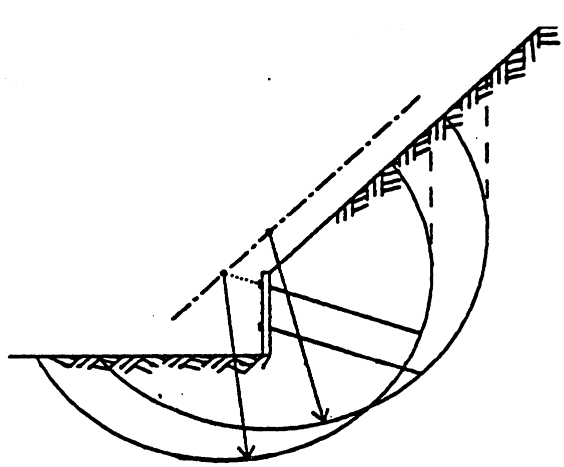
Δl_i = Length of the failure arc cut by the slice

L = Length of the entire failure arc

SPECIAL CONDITIONS

For major excavations in side slopes, slope stability failure for the entire system should be investigated.



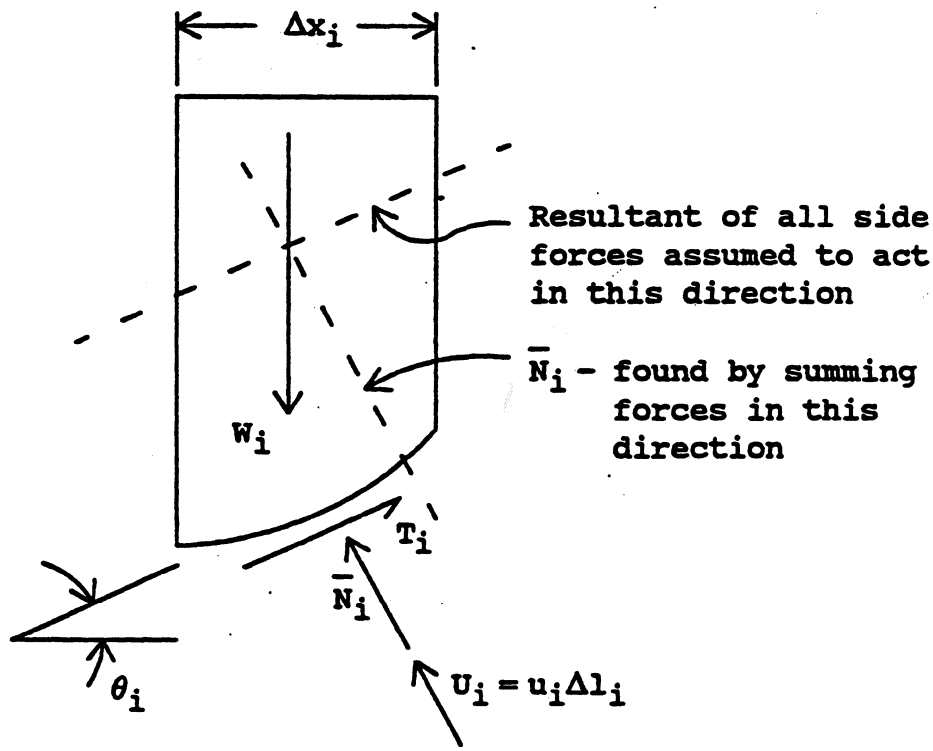


For major tie-back systems in other than optimum soils (as cohesive) over-all system failure should be investigated.

Minimum trial radii extend to end of ties; centers are on a line parallel to the slope.

FELLENIUS METHOD

Also known as 'Ordinary Method of Slices' or 'Swedish Circle'. This method assumes that for any slice, the forces acting upon its sides has a resultant of zero in the direction normal to the failure arc. This method errs on the safe side, but is widely used in practice because of its early origins and simplicity.



$$\bar{N}_i = W_i \cos\theta_i - u_i \Delta l_i$$

The basic equation becomes:

$$F = \{\bar{C}\,L + \tan\bar{\phi} \sum_{i=1}^{i=n} (W_i \cos\theta_i - u_i \Delta l_i)\} / \{\sum_{i=1}^{i=n} W_i \sin\theta_i\}$$

The procedure is to investigate many possible failure planes, with different centers and radii, to zero in on the most critical.

SAMPLE PROBLEM No. 20 - FELLENIUS METHOD OF SLICES

GIVEN:

γ = 115 pcf $\bar{\phi}$ = 30°   C = 200 psf   No groundwater

SOLUTION:

Scaled dimensions from graphic layout are satisfactory.

Angles

$\theta_1 = \sin^{-1}(6.7/60.0) = 6.41°$
$\theta_2 = \sin^{-1}(15.0/60.0) = 14.47$
$\theta_3 = \sin^{-1}(25.0/60.0 = 24.62$
$\theta_4 = \sin^{-1}(35.0/60.0) = 35.69$
$\theta_5 = \sin^{-1}(45.0/60.0) = 48.59$
$\theta_6 = \sin^{-1}(55.0/60.0) = 66.44$

Slice Weights

$\gamma = 0.115$ kcf
$W_1 = (1/2)(12.4)(10)(0.115) = 7.13k$
$W_2 = (17.8)(10)(0.115) = 20.47$
$W_3 = (27.6)(10)(0.115) = 31.74$
$W_4 = (34.7)(10)(0.115) = 39.91$
$W_5 = (40.0)(10)(0.115) = 46.00$
$W_6 = (35.8)(10)(0.115) = 41.17$

Slice	θ_i (°)	W_i (kips)	$W_i \sin\theta_i$	$W_i \cos\theta_i$	$\bar{N}_i$
1	6.41	7.13	0.80	7.09	7.09
2	14.47	20.47	5.11	19.82	19.82
3	24.62	31.74	13.22	28.85	28.85
4	35.69	39.91	23.28	32.41	32.41
5	48.59	46.00	34.50	30.43	30.43
6	66.44	41.17	37.74	16.46	16.46
			Σ=114.66		Σ=135.06

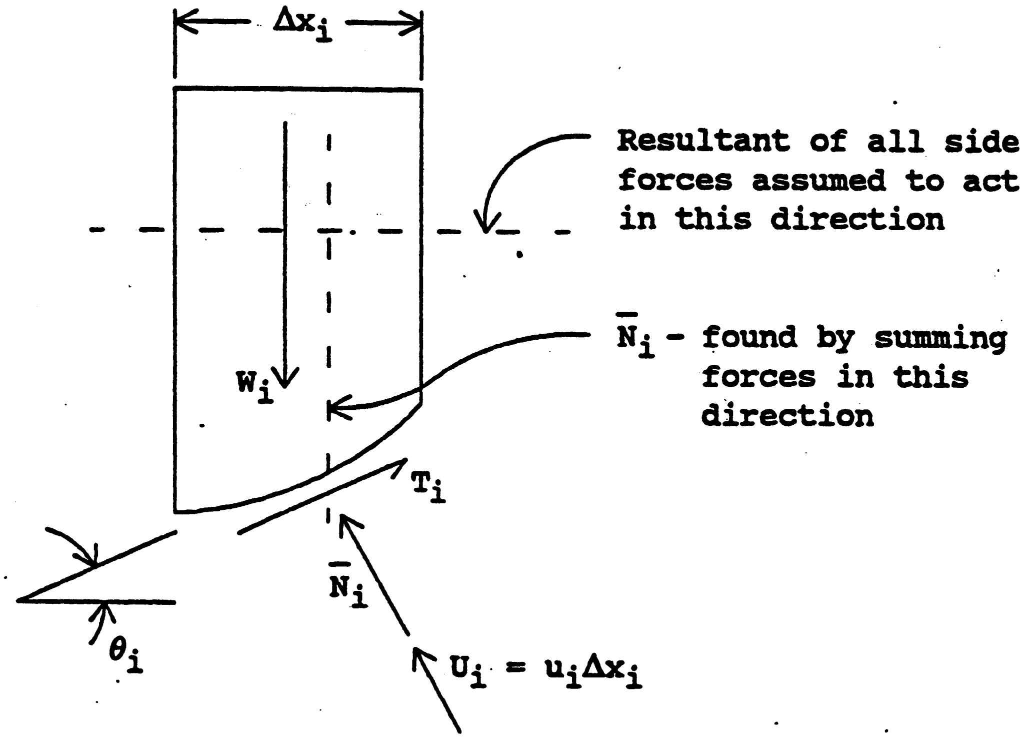
$U_i = 0$ L = 111.9'
(found by geometry & scaling)

$$F = \{(0.2)(111.9) + (0.577)(135.06)\}/114.66$$
$$= 0.87 < 1$$

This is the value for one trial failure plane. Additional trials are necessary to determine the critical one which gives minimum factor of safety. The slope for this sample problem is deemed to tie unstable since he computed safety factor determined by this single calculation is less than one.

BISHOP METHOD

This method, assumes that the forces acting on the sides of tiny slice have a zero resultant in the vertical direction.



$$\bar{N}_i = \{W_i - u_i \Delta x_i - (1/F_a)\bar{c}\Delta x_i \tan\theta_i\}/\cos\theta_i\{1 + (\tan\theta_i \tan\bar{\phi}\}/F_a$$

The basic equation becomes:

$$F = \{\sum_{i=n}^{i=n} (\bar{c}\ \Delta x_i + (W_i - u_i \Delta x_i)\tan\bar{\phi})(1/M_i)\}/\{\sum_{i=n}^{i=n} W_i \sin\theta_i\}$$

Where $\quad M_i = \cos\theta_i\{1 + (\tan\theta_i \tan\bar{\phi}/F_a)\}$

For Bishop Method, the Factors of Safety (F_a) must be assumed and a trial and error solution is required. The assumed "F_a's" converge on the Factor of safety for that trial failure plane. Good agreement between the assumed "F_a" and the calculated "F" indicates the selection of center and radius was good.

SAMPLE PROBLEM No. 21 - BISHOP METHOD

GIVEN:

Same as the previous example.

SOLUTION:

Column	A	B	C	D	E	F	G
Slice	θ_i	W_i	$\bar{C}\Delta x_i$	$W_i\tan\bar{\phi}$	$\cos\theta_i$	$\tan\theta_i\tan\bar{\phi}$	C + D
1	6.41	7.13	2.0	4.12	0.99	0.06	6.12
2	14.47	20.47	2.0	11.82	0.97	0.15	13.82
3	24.62	31.74	2.0	18.33	0.91	0.26	20.33
4	35.69	39.91	2.0	23.04	0.81	0.41	25.04
5	48.59	46.00	2.0	26.56	0.66	0.65	28.56
6	66.44	41.17	2.0	23.77	0.40	1.32	25.77

Column	Ha	Hb	Ia	Ib	J
Slice	M_i F_a=1.5	M_i F_a=0.8	G÷Ha F_a=1.5	G÷Hb F_a=0.8	$W_i\sin\theta_i$
1	1.04	1.07	5.94	5.72	0.80
2	1.06	1.15	12.92	12.02	5.11
3	1.07	1.21	19.00	16.94	13.22
4	1.04	1.23	24.31	20.36	23.28
5	0.95	1.20	30.06	23.80	34.50
6	0.75	1.06	34.36	24.31	37.74
			Σ=126.59	Σ=103.15	Σ=114.65

For F_a = 1.5
F=126.59/114.65 = 1.104

For F_a = 0.8
F=103.15/114.65 = 0.900

The factor of safety for this trial converges to ≈ 0.9.

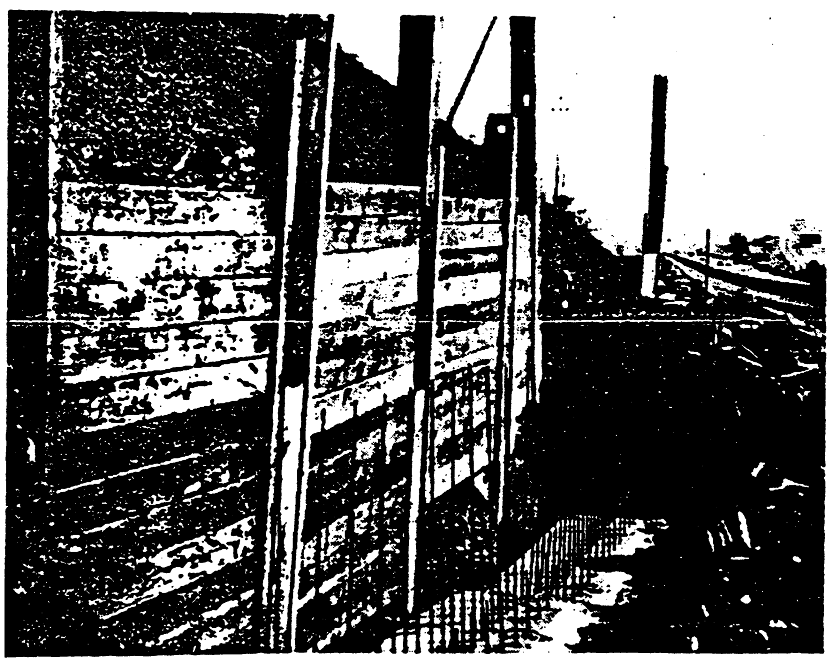
Again, this is the value for one trial failure plane. Additional trials are necessary to determine the critical one which gives minimum factor of safety.

If groundwater was present pore pressure would need to be considered. These values are most typically field measured.

The foregoing slope stability presentation serves to demonstrate the complexity of stability analysis. Soil failure analysis should not be limited to circular arc solutions. There are a number of computer programs for slope stability analysis using non-circular shapes. Slope stability analysis is most properly within the realm of geotechnical engineering.

When it appears that shoring or a cut slope presents a possibility of some form of slip failure, a stability analysis should be requested. In addition, the Transportation Materials and Research Laboratory in Sacramento has the capability of performing computer aided stability analysis to verify the submitted analysis.

Submittals relative to soils data and analysis should be from a recognized soils lab or from a qualified Geotechnical Engineer or Geologist.



HYDRAULIC FORCES ON COFFERDAMS AND OTHER STRUCTURES

Moving water imposes drag forces on obstructions in waterways. The drag force in equation form (after Ratay) is:

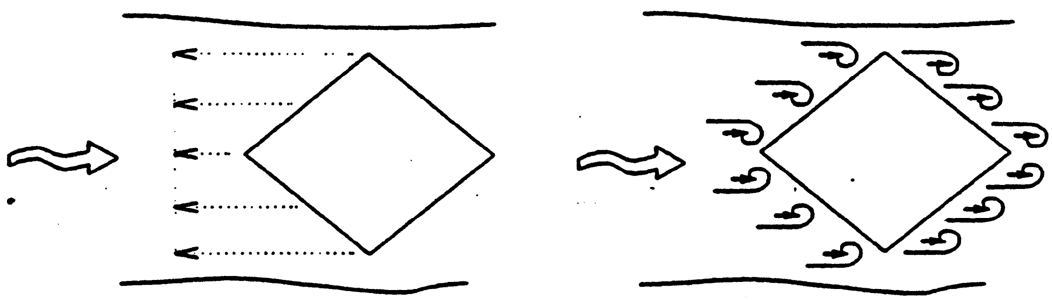
$$F_z = \rho(A)(C_d)(V^2/2g)$$

Where: ρ = Water density.
A = Projected area of the obstruction normal to the current.
C_d = Coefficient of drag.
V = Velocity of the current.
g = Acceleration due to gravity.

In english units $\rho \approx 2g$ so that:

$$F_z = A(C_d)(V^2)$$

Where: A = Ft^2
C_d = pounds
V = ft/sec.



Considering roughness along the sides of the obstructions (as for a sheetpile cofferdam the practical value for C_d = 2.0.

$$F_z = 2AV^2$$

Which may be considered to be applied in the same manner as a wind rectangular load on the loaded height of the obstruction.

Example: Determine the drag force on a six foot diameter corrugated metal pipe placed vertically in water of average depth of 6 feet flowing at 4 feet per second.

Projected Area = 6(6) = 36 Ft^2.

F_Z = 2{36)(4)^2 = 1,152 Lbs.